Municipal Corp. of
Cape Charles

June 11, 2009

Mr. John M. Kennedy
Department of Environmental Quality
Office of Water Quality Programs
PO Box 1105
Richmond, VA 23218

Dear Mr. Kennedy,

Per Ms. Gilinsky's letter of May 8, 2009, attached is our petition to the State Water Control Board to extend the deadline to December 31, 2015 to operate at the higher design flow as indicated in 9 VAC 25-720. Should you have any questions or need additional information, I can be reached at 757-331-3259, x19, or bob.panek@capecharles.org.

Sincerely,

Robert L. Panek

Enclosures

Municipal Building • 2 Plum Street • Cape Charles, Virginia 23310
(757) 331-3259 Fax (757) 331-4820

June 11, 2009

To: State Water Control Board

From: Town of Cape Charles
2 Plum St.
Cape Charles, VA 23310
757-331-3259

Petition for Amendments to Nutrient Waste Load Allocations

The Town of Cape Charles petitions the State Water Control Board to amend the nutrient waste load allocation in 9 VAC 25-720 applicable to the Cape Charles Wastewater Treatment Plant. Specifically, we request extension of the deadline to December 31, 2015 to operate and discharge at the higher design flow of 500,000 gallons per day. Our town has some unique characteristics, primarily related to our evolution into a resort and retirement community, that support this request.

We currently have about 1,000 full-time residents (1,500 with part-timers) that produce an average daily wastewater flow of about 140,000 gallons per day (GPD), about 55% of the capacity of our current 250,000 GPD plant Based on our low average household water consumption of 120 GPD, this equates to about 1,200 Equivalent Residential Connections (ERCs). However, we have three approved large mixed use developments that will contribute to significant future growth. Substantial development activity at any one of the three would quickly push our current plant towards capacity. If ultimately built-out, we will see growth to over 5,500 ERCs and an average daily flow of about 700,000 GPD. Our February 2009 growth projection is attached.

Because of this projected growth, we had previously planned to build a higher capacity 500,000 GPD Membrane Bio Reactor plant that would comply with the nutrient waste load allocations. Our Preliminary Engineering Report (PER) was approved by the Department of Environmental Quality (DEQ) on July 17, 2008. The pace of growth previously anticipated would have maximized our current 250,000 GPD capacity by about 2013. However, the economic recession has slowed the pace considerably. Assuming economic recovery by the end of 2010, we now anticipate 250,000 GPD to be viable until about 2016. Given the uncertain timing of economic recovery, the Town opted to reduce the replacement plant from 500,000 to 250,000 GPD to minimize underutilized capacity and the larger capital costs that would become a great burden on our existing customers. Further details are provided in the attached PER Addendum of March 9, 2009. Drawing M-02, providing design criteria, is also attached.

Many aspects of the 250,000 GPD plant we are designing, such as site development, utilities and some structures and systems (preliminary treatment, disinfection, etc), are the same as for the larger capacity plant. Additionally, site layout, power distribution and process piping are being designed to accommodate future expansion. We are therefore positioning the Town for an easier expansion to 500,000 GPD when the growth does occur. This, of course, comes at a higher cost now than if we were to build a 250,000 GPD plant with no anticipation of future growth. Drawing C-03, showing provisions for expansion, is attached.

In light of this, we would like to avoid prematurely incurring the added capital cost associated with effluent reuse by preserving the 500,000 GPD waste load allocation. We recognize that effluent reuse will be required at some time in the future, even with a 500,000 GPD waste load allocation, if our growth projections are ultimately realized. However, we would like to pace implementation with the substantial accumulation of facility fees from new connections so we do not place an additional financial burden on our customers. In the interim, the Membrane Bio Reactor system we are building will produce effluent of the highest quality that is expected to exceed waste load standards.

Extension of the deadline to December 15, 2015 to operate at the 500,000 GPD design flow will afford us the opportunity to:

1. Further evaluate the prospects for economic recovery and accelerated growth.
2. Expand the capacity of our replacement plant either during construction or soon after completion in 2011 if we are confident that growth will materialize.
3. Delay the added capital cost of implementing effluent reuse until we are better able to afford it.

Dora Sullivan
Mayor

TIME PERIOD	% GROWTH	Bay Creek - South Tract UNITS / YEAR	AMENITY / YEAR	TOTAL PER YEAR	CUMULATIVE ERC	Historic District HIST DIST	CAPE HARBOR	SOUTH PORT	HARBOR DEV GRP	KECK	BC Marina Residential	Marina Amenity	TOTAL PER YEAR	CUMULATIVE ERC
As of Sep 2008		250	8	258	258	855					68	64	987	987
2009	0.0%	0	0	0	258	0		10			0	0	10	997
2010	1.0%	24	0	24	282	2	0	0	0	0	2	3	7	1,004
2011	1.5%	36	0	36	319	2	0	10	0	0	4	3	19	1,023
2012	1.8%	43	20	63	381	2	55		100	0	4	3	164	1,187
2013	1.8%	43	20	63	444	3	0	20	0	12	4	6	45	1,232
2014	1.8%	43	20	63	506	3	55		50	0	4	6	118	1,351
2015	2.0%	49	20	69	575	3	0	20	0	12	5	4	44	1,394
2016	2.0%	49	20	69	644	3	175		102	0	5	50	335	1,729
2017	2.0%	49	20	69	712	3	0	20	0	12	5	3	43	1,772
2018	2.0%	49	20	69	781	3	65		52	12	5	3	140	1,912
2019	2.5%	61	20	81	862	3	0	20	0	12	6		41	1,953
2020	3.5%	85	30	115	977	3	65		100	12	8		188	2,141
2021	4.0%	97	40	137	1,114	4	0			12	10		26	2,167
2022	4.5%	109	40	149	1,264	4	60		100	12	11		187	2,354
2023	4.5%	109	40	149	1,413	4					11		15	2,368
2024	5.0%	124	40	164	1,577	4					12		16	2,384
2025	5.0%	124	40	164	1,740	4					12		16	2,400
2026	5.0%	124	40	164	1,904	4					12		16	2,416
2027	5.0%	124	22	146	2,050	4					12		16	2,432
2028	5.0%	124	0	124	2,173	4					12		16	2,448
2029	5.0%	124	0	124	2,297	3					12		15	2,463
2030	4.5%	109	0	109	2,406	3					11		14	2,477
2031	4.5%	109	0	109	2,516	2					11		13	2,490
2032	4.0%	97	0	97	2,613						10		10	2,500
2033	4.0%	97	0	97	2,710						10		10	2,509
2034	4.0%	97	0	97	2,807						10		10	2,519
2035	4.0%	97	0	97	2,905						9		9	2,527
2036	4.0%	97	0	97	3,002						9		9	2,536
2037	3.5%	85	0	85	3,087						7		7	2,543
2038	2.8%	67	0	67	3,154						7		7	2,550
	100.0%	2,694	460	3,154		925	475	100	504	96	305	145	2550	

TIME PERIOD	CUM TOTAL	250gpd / ERC	225gpd / ERC	200gpd / ERC	175gpd / ERC	150gpd / ERC	125gpd / ERC	120gpd / ERC
As of Sep 2008	1,245	311,250	280,125	249,000	217,875	186,750	155,625	149,400
2009	1,255	313,750	282,375	251,000	219,625	188,250	156,875	150,600
2010	1,287	321,680	289,512	257,344	225,176	193,008	160,840	154,406
2011	1,342	335,450	301,905	268,360	234,815	201,270	167,725	161,016
2012	1,569	392,140	352,926	313,712	274,498	235,284	196,070	188,227
2013	1,676	419,080	377,172	335,264	293,356	251,448	209,540	201,158
2014	1,857	464,270	417,843	371,416	324,989	278,562	232,135	222,850
2015	1,970	492,380	443,142	393,904	344,666	295,428	246,190	236,342
2016	2,373	593,240	533,916	474,592	415,268	355,944	296,620	284,755
2017	2,484	621,100	558,990	496,880	434,770	372,660	310,550	298,128
2018	2,693	673,210	605,889	538,568	471,247	403,926	336,605	323,141
2019	2,815	703,660	633,294	562,928	492,562	422,196	351,830	337,757
2020	3,118	779,540	701,586	623,632	545,678	467,724	389,770	374,179
2021	3,281	820,260	738,234	656,208	574,182	492,156	410,130	393,725
2022	3,617	904,320	813,888	723,456	633,024	542,592	452,160	434,074
2023	3,782	945,380	850,842	756,304	661,766	567,228	472,690	453,782
2024	3,961	990,280	891,252	792,224	693,196	594,168	495,140	475,334
2025	4,141	1,035,180	931,662	828,144	724,626	621,108	517,590	496,886
2026	4,320	1,080,080	972,072	864,064	756,056	648,048	540,040	518,438
2027	4,482	1,120,480	1,008,432	896,384	784,336	672,288	560,240	537,830
2028	4,622	1,155,380	1,039,842	924,304	808,766	693,228	577,690	554,582
2029	4,760	1,190,030	1,071,027	952,024	833,021	714,018	595,015	571,214
2030	4,883	1,220,840	1,098,756	976,672	854,588	732,504	610,420	586,003
2031	5,006	1,251,400	1,126,260	1,001,120	875,980	750,840	625,700	600,672
2032	5,112	1,278,120	1,150,308	1,022,496	894,684	766,872	639,060	613,498
2033	5,219	1,304,840	1,174,356	1,043,872	913,388	782,904	652,420	626,323
2034	5,326	1,331,560	1,198,404	1,065,248	932,092	798,936	665,780	639,149
2035	5,432	1,358,030	1,222,227	1,086,424	950,621	814,818	679,015	651,854
2036	5,538	1,384,500	1,246,050	1,107,600	969,150	830,700	692,250	664,560
2037	5,631	1,407,630	1,266,867	1,126,104	985,341	844,578	703,815	675,662
2038	5,704	1,426,000	1,283,400	1,140,800	998,200	855,600	713,000	684,480
	5,704							

NOTES / ASSUMPTIONS:

1. Up to 3,000 residential units allowed for Bay Creek.
2. 318 residential units have been constructed as of September 2008 - 68 North and 250 South
3. Additional build out period assumed to be 30 years starting in 2009
4. Percent growth is the percent of total units to be built in a specific year
5. 38 Island Villas included in Marina Amenities. Marina hotel is 46 ERC in 2016.
6. Water use for future south side amenities is projected to be equivalent to 460 ERC. This figure excludes pools, fountains, outdoor showers, HVAC, etc for the Beach Club and the Inn & Spa.
7. Future amenities assumed to be built in 16 years, starting 2012 roughly keyed to residential growth
8. Lot sell out assumed to occur 4 to 5 years prior to build out

Cape Harbor assumes the developers request, reduced by 125 residential units per Council action
South Port reflects a "best guess" in absence of a development plan.
Harbor Development Group reflects the approved conditional use permit.
Keck assumes vitually all of the request under the approved rezoning.

STEARNS & WHELER
CLIENTS|PEOPLE|PERFORMANCE

TECHNICAL MEMORANDUM

To: Marcia Degan, Virginia Department of Environmental Quality

From: Stearns and Wheler LLC

Date: March 9, 2009

Re: PER Addendum
Cape Charles WWTP Nutrient Removal Upgrade
81168.5

1.0 PURPOSE

The Preliminary Engineering Report (PER) for the Town of Cape Charles Wastewater Treatment Plant (WWTP) Improvements were issued to DEQ on June 17, 2008. The PER was approved by DEQ on July 17, 2008.

This Addendum will act as a summary of design modifications that will be implemented as a result of the Value Engineering session held in December 2009 (Section 2 of this submittal) as well as the decision to reduce the design capacity of the proposed WWTP from 0.5 mgd to 0.25 mgd made by the Town of Cape Charles.

2.0 FLOW PROJECTIONS

The growth projections presented in Chapter 2 of the PER were completed prior to the economic downturn and resulted in rapid development of the service area. The expansion of the WWTP to 0.5 mgd was driven both by these growth projections as well as regulatory requirements to maintain the foot-noted waste load allocation. The Town re-estimated growth projections in February 2009 in recognition of the current economic recession. This led to the decision to down-size the plant capacity from 0.5 to 0.25 mgd. The graph presented in Figure 2-1 shows the following growth projections: PER (June 2008), VE (November 2008) and current (February 2009) at 156 gpd/EDU and 120 gpd/EDU. The most recent data indicate an average demand of 120 gpd. This is the result of both water conservation and inflow and infiltration reduction efforts.

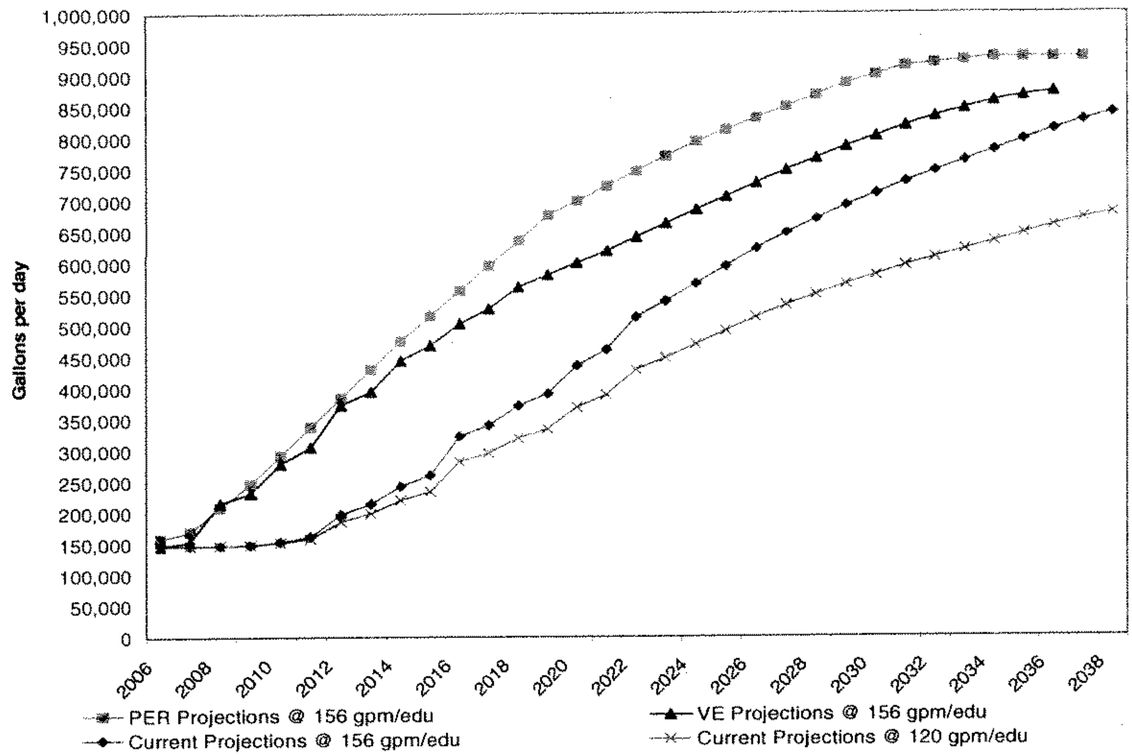

Figure 2-1: Growth Projections for the Town of Cape Charles and Bay Creek.

The new growth projections are presented in Figure 2-2. These projections assume recovery from the economic recession by the end of 2010, followed by phased construction of two approved large mixed use developments beginning in 2011. Additionally, the projections also reflect improvement in the building rate in the Bay Creek PUD after 2010. Figure 2-2 also displays reduced growth curves representing 25% and 50% of the expected growth. The reduced growth curves allow an understanding of the range of expected capacity lifetimes and future phases.

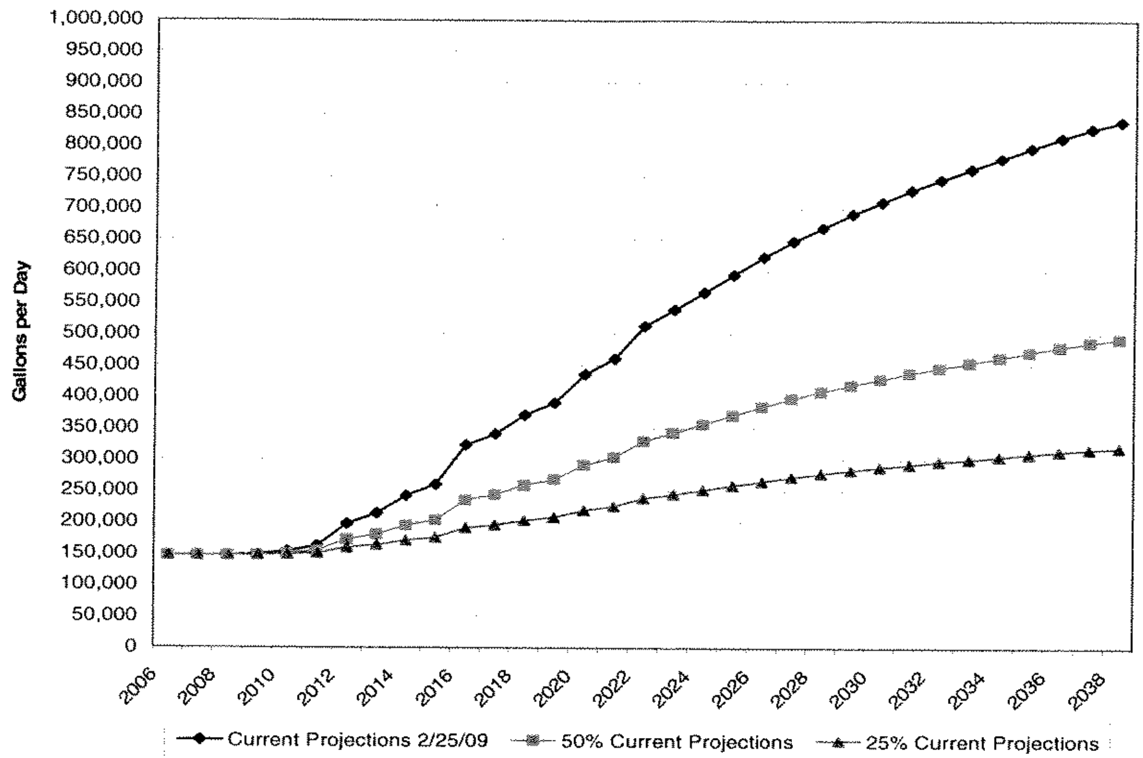

Figure 2-2: Potential Growth Projections Based on Current Growth Projections for the Town of Cape Charles and Bay Creek.

As shown in Figure 2-2, based on the current growth projections the Town of Cape Charles WWTP would achieve 0.25 mgd in 2015. However, due to the uncertainties associated with predicting the timing of recovery from the current economic recession, wastewater flows may not reach the plant capacity of 0.25 mgd until 2025 if the service area grows at a rate of 25% of the projected growth rate. The Town anticipates 10 new connections in 2009; however, the Town has not received a request for new connection since 2007.

Hence, since the recovery of the economy is such a significant factor in the Town's required treatment capacity, it was deemed appropriate by the Town to size a plant for 0.25 mgd, easily expandable in the future to 0.5 mgd. Provisions for water reuse will be necessary for the 0.5 mgd expansion due to the capped waste load allocation.

3.0 DESIGN FLOW AND LOADS

The PER design loads were based on an average daily design flow of 0.5 mgd. Peak hour flows were developed based on a 4:1 peaking factor based on a review of available historical data and projections for new growth. A review of more recent plant flow data has resulted in an updated peak hour flow. Design flows and loads for the 0.5 mgd facility are presented in Tables 3-1 and 3-2. Table 3-3 and 3-4, summarize the design flows and loads for a facility with an average daily flow of 0.25 mgd.

Table 3-1: PER Design Flow Rates

CONDITION	FLOW (MGD)	PEAKING FACTOR
Average Daily Flow	0.5	N/A
Average Daily Flow with Recycles(1)	0.52	N/A
Maximum Month Flow with Recycles(2)	0.77	1.5
Maximum Day Flow with Recycles(3)	1.32	2.6
Peak Hour Flow with Recycles(4)	2.03	3.9
Peak Hour Flow (after Flow Equalization)(5)	1.58	3.0

Notes:

1. Recycle flows estimated to be 0.02 mgd.
2. Maximum month flow based on: 0.5 mgd * 1.5 + 0.02 mgd (estimated recycle flows from future nutrient removal process).
3. Maximum day flow based on: 0.5 mgd * 2.6 + 0.02 mgd (estimated recycle flows from future nutrient removal process).
4. Peak hour flow based on: 0.5 mgd * 3.9 + 0.08 mgd (estimated recycle flows from future nutrient removal process).
5. Refer to Section 5 of the PER Addendum for Flow Equalization discussion.

Table 3-2: PER Design Loads

PARAMETER	AVERAGE CONCENTRATION (MG/L)(3)	AVERAGE LOAD (LBS/D)(1,5)	MAXIMUM MONTH CONCENTRATION (MG/L)(4)	MAXIMUM MONTH LOAD (LBS/D)(2,5)
Flow, mgd	0.52	0.52	0.52	0.52
BOD	218	944	283	1,228
COD	495	2,145	643	2,789
TSS	240	1,039	311	1,351
TKN	42.0	182	54.6	237
NH_4-N	33.2	144	43.1	187
TP	6.5	28.1	8.4	36.5
Ortho-P	5.0	21.6	6.5	28.1
Alkalinity	283	1,225	367	1,593

Notes:

1. Average load based on existing average load at 0.15 mgd and future average load at 0.35 mgd
2. Maximum load based on maximum month load at 0.15 and future maximum month load at 0.35 mgd.
3. Average concentration based on average load (lb/d) / 0.5 mgd / 8.34 (conversion factor).
4. Maximum month concentration based on maximum month load (lb/d) / 0.5 mgd / 8.34 (conversion factor).
5. Average and maximum month loads do not include solids process recycles.

Table 3-3: 0.25 mgd WWTP Design Flow Rates

PARAMETER	FLOW (MGD)	PEAKING FACTOR
Average Daily Flow	0.25	N/A
Average Daily Flow with Recycles (1)	0.28	N/A
Maximum Month Flow with Recycles(2)	0.40	1.5
Maximum Day Flow with Recycles(3)	0.68	2.6
Peak Hour with Recycles(4,5)	0.78	3.0

Notes:
1. Recycle flows estimated to be 0.03 mgd.
2. Maximum month flow based on: 025 mgd * 1.5 + 0.03 mgd (estimated recycle flows from future nutrient removal process).
3. Maximum day flow based on: 0.25 mgd * 2.6 + 0.03 mgd (estimated recycle flows from future nutrient removal process).
4. Peak hour flow based on: 0.2.5 mgd * 3.0 + 0.03 mgd (estimated recycle flows from future nutrient removal process).
5. Refer to Section 5.1 of the PER Addendum for Flow Equalization discussion.

Table 3-4: 0.25 mgd WWTP Design Loads

PARAMETER	AVERAGE CONCENTRATION (MG/L)	AVERAGE LOAD (LBS/D)(1)	MAXIMUM MONTH CONCENTRATION (MG/L)	MAXIMUM MONTH LOAD (LBS/D)(1)
Flow, mgd	0.28	0.28	0.28	0.28
BOD	206	472	268	614
COD	468	1,073	608	1,394
TSS	226	519	294	675
TKN	39.7	91	51.6	118
NH_4-N	31.4	72	40.8	94
TP	6.1	14	8.0	18
Ortho-P	4.7	11	6.1	14
Alkalinity	267	613	347	796

Notes:
1. Average and maximum month loads based on 50% of the original design loads.

4.0 NUTRIENT REMOVAL ALTERNATIVES

4.1 Biological Reactors

Per Section 4 of the PER, the membrane bioreactor (MBR) was selected for the new Town of Cape Charles WWTP. As a part of the MBR, two (2) 5-Stage Bardenpho reactors were recommended as the biological process. Table 4.1-1 lists the biological reactor design criteria for the 5-Stage Bardenpho reactors for the 0.5 mgd facility.

Table 4.1-1: PER Reactor Design Criteria

PARAMETER	HRT(1) (HRS)	VOLUME (MG)	VOLUME PER REACTOR (MG)
Anaerobic Zone	1.0	20,000	10,000
Pre-Anoxic Zone	1.0	20,000	10,000
Pre-Anoxic Zone	1.0	20,000	10,000
Aerobic Zone	5.7	120,000	60,000
Post-anoxic Zone	1.4	30,000	15,000
Post-anoxic Zone	1.0	20,000	10,000
Total	11.0	230,000	115,000

Note:
1. HRT calculated based on a nominal average daily flow of 0.5 mgd.

VE Modification

The proposed 5-Stage Bardenpho process was modified to a 4-Stage Bardenpho process as a result of the VE session.

Current Direction

Two (2) 4-Stage Bardenpho reactors will be provided to achieve the required nitrogen and phosphorus limits. Table 4.1-2 summarizes the preliminary design criteria for the reactors.

Table 4.1-2: 0.25 mgd Reactor Design Criteria

PARAMETER	HRT(1) (HRS)	VOLUME (MG)	VOLUME PER REACTOR (MG)
Pre-Anoxic Zone	1.44	15,000	7,500
Pre-Anoxic Zone	1.44	15,000	7,500
Aerobic Zone	4.32	45,000	22,500
Aerobic Zone	4.32	45,000	22,500
Post-anoxic Zone	1.44	15,000	7,500
Post-anoxic Zone	1.44	15,000	7,500
Total	14.4	150,000	75,000

Note:
1. HRT calculated based on a nominal average daily flow of 0.25 mgd.

4.2 Membrane Tanks

As a part of the MBR, three (3) membrane tanks were proposed downstream of the biological reactors for solids separation. Table 4.2-1 lists the membrane filtration system design criteria for the 0.5 mgd facility.

Table 4.2-1: PER Membrane Filtration System Design Criteria

UNIT PROCESS / EQUIPMENT	VALUE
Membrane Tanks(1)	
No. of Trains	3
No. of Installed Cassettes / Train	2
Total No. of Cassettes / Train	3
Type of Cassette	ZeeWeed 500d
Design MLSS	8,000 – 10,000 mg/L
No. of Modules / Cassette	44
Total No. of Modules	264
Pore Size	0.04 microns (ultrafiltration)
Design Average Daily Flow(2)	0.52 mgd
Design Maximum Month Flow(2)	0.77 mgd
Design Peak Daily Flow(2)	1.58 mgd
Design Peak Hourly Flow(2)	2.00 mgd
N-1 Condition (N=1 membrane tank)	2.00 mgd

Notes:
1. Recommendations are based on Zenon proposal dated August 9, 2007.
2. Flows include plant internal recycle flows.

VE Modification

No modifications were proposed to the membrane filtration system as a result of the VE session.

Current Direction

Table 4.2-2 summarizes the membrane filtration system design criteria used prior to the VE session and the membrane filtration design criteria for the 0.25 mgd WWTP.

Table 4.2-2: VE and Current Membrane Filtration System Design Criteria

UNIT PROCESS / EQUIPMENT	VE VALUE	CURRENT VALUE
Membrane Tanks(1)		
No. of Trains	3	2
No. of Installed Cassettes / Train	2	3
Total No. of Cassettes / Train	3	3
Type of Cassette	ZeeWeed 500d	ZeeWeed 500d
Design MLSS	8,000 – 10,000 mg/L	8,000 – 10,000 mg/L
No. of Modules / Cassette	44	34 (4 cassettes) 36 (2 cassettes)(1)
Total No. of Modules	264	208(1)
Pore Size	0.04 microns (ultrafiltration)	0.04 microns (ultrafiltration)
Design Average Daily Flow(2)	0.55 mgd	0.28 mgd
Design Maximum Month Flow(2)	0.8 mgd	0.40 mgd
Design Peak Daily Flow(2)	1.5 mgd	0.68 mgd
Design Peak Hourly Flow(2)	1.5 mgd	0.78 mgd
N-1 Condition (N=1 membrane tank)	1.5 mgd	0.78 mgd

Notes:
1. Recommendations are based on a Zenon proposal dated March 4, 2009.
2. Flows include plant internal recycle flows.

5.0 UNIT PROCESSES

5.1 Flow Equalization

Per Section 6.4 of the PER, one (1) 440,000 gallon flow equalization tank comprising of two (2) compartments was recommended for the new WWTP to reduce the peak hour flow to average daily flow ratio from 3.9 to 1 to 3.0 to 1 and to maximize WQIF Grant eligibility. The peak hour ratio of 3.9 to 1 was developed based historic plant data and estimates for the new collection system. Upon investigation of the original chart records (2006), a peak hour flow to average daily flow ratio of 6 to 1 was developed for the existing collection system associated with an average plant influent flow of approximately 0.15 mgd. The flow associated with growth (0.35 mgd) was anticipated from new development and a peak hour ratio to average daily flow of 3.0 to 1 was assumed. A weighted average of the existing peak hour ratio (3.9 to 1) and the new development peak hour factor (3.0 to 1) which resulted in plant peak hour ratio of 3.9 to 1.

VE Modification

During the VE session, the VE team questioned the need for flow equalization, because the plant start-up flows will be significantly below design capacity and the peak hour flow to average daily flow ratio of 6 to 1 for the existing collection system seemed excessive (installed in mid-1980s). As a result, the Design Team reviewed the peak hour flow data for the existing system.

In addition, since 2006 the Town has undertaken a significant I&I reduction program, ranging from I&I studies, public awareness, CCTV work, smoke-testing, correction of illegal connections, and an overall tightening on water consumption. Not only has the overall water consumption (per EDU) dropped by approximately 20% since 2006, but there have been only two (2) overflows within the Town in the last 18 months (12/08 and 7/07 which were related to pump failures which the Town is correcting). Based on a review of the 2008 wet weather days, no sustained peak hour flows were recorded in excess of 3 to 1. The few peaks in excess of a 3 to 1 ratio were instantaneous and no longer than a few minutes in duration, attributable to the constant speed, low flow, pump stations from Bay Creek.

Therefore, it was determined that flow equalization was not required for Phase I because the liquid treatment train was designed to handle a peak hour ratio of 3 to 1. As an alternative, a third reactor was added to the design to serve as an emergency overflow tank that could be used for off-line storage during a plant upset or key equipment failure.

Current Direction

A flow equalization system will not be provided as a part of the current design. However, an emergency overflow tank will be provided with a minimum of 12 hours of storage at an average daily flow rate of 0.25 mgd. Site provisions will be made to allow for construction of future FEQ tanks in the event that peaking factors change over time.

5.2 Headworks Facility

Per Section 5.2.4 of the PER, an enclosed Headworks Facility was recommended for the new WWTP. The following summarizes the recommended Headworks Facility for the 0.5 mgd WWTP.

- One (1) 6-mm automatic screen for coarse screening
- One (1) 6-mm manually cleaned bar rack for emergency bypass
- One (1) vortex grit tank for grit removal
- Two (2) 2-mm automatic screens for fine screening (required for MBR)
- One (1) Parshall flume for flow measurement.

The Headworks was designed to treat the future average design flow of 1.0 mgd and was sized to handle a future peak flow of 4.06 mgd.

VE Modification

The 6-mm coarse screen was removed from the Headworks Facility as a result of a VE recommendation. A second 6-mm manually cleaned bar rack was added to the Headworks Facility. In addition the Headworks Facility footprint was reduced due to the deletion of flow equalization from Phase I as discussed in Section 5.1.

Current Direction

The Headworks Facility will be sized to treat the future design average daily flow of 0.5 mgd and the associated peak hour flow of 1.5 mgd. The reduction in design flows will result in smaller mechanical equipment.

5.3 Disinfection

PER Recommendation

Per Section 5.3.2 of the PER, a new UV disinfection system was recommended for the WWTP. The UV system design criteria are listed in Table 5.3-1.

Table 5.3-1:PER UV Disinfection System Design Criteria

PARAMETER	UNITS	DESIGN CRITERIA
UV Transmissivity(1)	%	65
Maximum TSS through UV(2)	mg/L	10
Required Downstream Fecal Coliforms(3)	N/CML	200
No. of Banks	N/A	3 (2+1 standby)
Average Daily Flow Treated/Bank(1)	mgd	Phase I: 0.5 Phase II:1.0
Peak Hourly Flow Treated/Bank(1)	mgd	Phase I: 2.0 Phase II: 4.0
No. of Modules per Bank(1)	N/A	Phase I: 2 Phase II: 4
Total Number of UV Lamps(1)	N/A	Phase I: 24 Phase II: 48
Peak Power Requirements(1)	Max kW / unit	Current-6 kW
Required Channel Length(1)	ft.	40
Required Channel Width(1)	in.	16

Notes:
1. Recommendations above are based on the Trojan UV3000Plus System.
2. Anticipated effluent quality from membrane or effluent filtration process.
3. Based on existing NPDES permit.

VE Modification

No changes to the UV disinfection system were recommended as part of the VE process except optimization of the channel layout.

Current Direction

Table 5.3-2 below lists the updated design criteria for the UV disinfection system for the 0.25 mgd WWTP.

Table 5.3-2: Current UV Disinfection System Design Criteria

PARAMETER	UNITS	DESIGN CRITERIA
UV Transmissivity[1]	%	75
Maximum TSS through UV[2]	mg/L	10
Required Downstream Fecal Coliforms[3]	N/CML	200
No. of Banks	N/A	2 (1+1 standby)
Average Daily Flow Treated/Bank[1]	mgd	Phase I: 0.25 Phase II:0.5
Peak Hourly Flow Treated/Bank[1]	mgd	Phase I: 0.75 Phase II: 1.5
No. of Modules per Bank[1]	N/A	Phase I: 2 Phase II: 3
Total Number of UV Lamps[1]	N/A	Phase I: 16 Phase II: 24
Peak Power Requirements[1]	Max kW / unit	Current-6 kW
Required Channel Length[1]	ft.	40
Required Channel Width[1]	in.	12

Notes:
1. Recommendations above are based on the Trojan UV3000Plus System.
2. Anticipated effluent quality from membrane or effluent filtration process.
3. Based on existing NPDES permit.

5.4 Post Aeration

PER Recommendation

Per Section 5.4.2 of the PER, a diffused aeration system was recommended for post aeration. However, the PER was developed based on the concept of constructing the facility at the site of the existing WWTP. Therefore, the diffused aeration system was included in an existing structure. Following the completion of the PER, it was decided that the new WWTP would occupy a greenfield site adjacent to the existing WWTP. The elevation of the new site made it hydraulically feasible to implement cascade aeration as a means of post aeration which would result in reduced plant operating costs. The cascade aeration system will be designed to meet the effluent dissolved oxygen concentration. The system will be designed with an average loading rate of 0.25 mgd/ft. As a result, the width of the steps will be 5 feet to handle the average daily flow of 0.5 mgd. There will be five (5) steps and with a tread of 1'-1" per step.

VE Modification

No changes to the cascade aeration system were recommended as part of the VE process except optimization of the cascade aeration and UV system layout.

Current Direction

The cascade aeration system will be designed to meet the effluent dissolved oxygen concentration. The system will be designed with an average loading rate of 0.25 mgd/ft. As a result, the width of the steps will be reduced from 5 feet to 3 feet to handle the average daily flow of 0.25 mgd. The number of steps and the tread of the steps will remain the same.

5.5 Effluent Flow Measurement

PER Recommendation

Per Section 5.5 of the PER, a new 9-inch Parshall Flume was to be provided to measure the design effluent flow rates associated with the Phase I 0.5 mgd design flow rates and the Phase II 1.0 mgd design flow rates.

VE Modification

No changes were proposed to the effluent measurement as part of the VE process.

Current Direction

The Parshall Flume will be reduced from a 9-inch to a 6-inch throat which is sufficient to handle the Phase I design flow rates associated with the 0.25 mgd plant and the Phase II design flow rates associated with the 0.5 mgd plant.

5.6 Chemical Feed Systems

PER Recommendation

Per Section 5.6 of the PER, methanol was proposed as the supplemental carbon source required for denitrification and ferric chloride was proposed as the metal coagulant required for chemical phosphorus removal. During final design it was decided that a variety of supplemental carbon sources, including methanol, should be considered for denitrification. Additionally, based on the current usage of ferric chloride for phosphorus removal as part of the Interim Optimization Plan, the Town indicated that alum was the preferred metal coagulant.

VE Modification

As part of the VE process, use of methanol was removed from the list of potential supplemental carbon sources in an effort to reduce the feed facility cost and hazards of the supplemental carbon storage facility.

Current Direction

A non-hazardous supplemental carbon source (i.e. MicroC-G, sugar water, glycerin, etc) will be used for denitrification. The supplemental carbon feed facility will be designed to accommodate multiple non-hazardous supplement carbon source. An alum feed facility will be provided for chemical for phosphorus removal. Adequate storage volume will be provided for both chemicals.

5.7 Electrical Power Distribution Needs

PER Recommendation

Per Section 5.7 of the PER, the replacement of existing electrical facilities was recommended to ensure reliable operation for a minimum of 20 years. As a result, a new feeder would be required as well as an emergency generator system to provide adequate emergency power. The power system was sized to accommodate future Phase II electrical loads. During the final design process, a double ended main–tie-main power distribution was recommended for redundancy and maximum reliability.

VE Modification

The double ended power distribution was replaced with a simple radial feed system as a result of the VE session.

Current Direction

New electrical facilities will be provided to ensure reliable operation for a minimum of 20 years. The power distribution system will be radial feed. An emergency generator will be provided for emergency power generation.

5.8 Process Control System

PER Recommendation

A plant wide process control system was recommended by integrating control automation in order to enhance daily operations and overall facility performance. The installation of a PLC (programmable logic controller) based system was recommended as a distributed control system.

VE Recommendation

No changes were proposed to the process control system during the VE session.

Current Direction

A plant wide process control system will be provided for the 0.25 mgd WWTP.

5.9 Hydraulic Profile

PER Recommendation

As discussed in PER Section 7, raw wastewater will be pumped from the collection system via two (2) main wastewater pump stations to the Headworks facility. Wastewater will flow by gravity through the screens and grit removal in the Headworks Facility, through the distribution structures, reactors, and into membrane tanks. Permeate pumps will draw treated water through the membranes and into the backpulse tank. Permeate will flow by gravity from the backpulse tank to the UV system, post aeration, effluent flow measurement, through the outfall and into the Chesapeake Bay.

The WWTP will be designed to hydraulically pass the peak instantaneous flow without flow equalization; however, the treatment processes will be designed for the peak hour flow. Therefore, the top of wall elevations for the process structures and manholes will be designed for peak instantaneous flow conditions.

The bottom elevation of the last step in the cascade aeration basin will be set such that it is not submerged during peak flow conditions at the high tide elevation in the bay. The plant outfall will be evaluated based on the average and peak daily design flows at a high tide elevation of 5.83 feet and the 100-year flood elevation of 9.00 feet.

VE Modification

No changes were proposed to the hydraulic profile during the VE session.

Current Direction

The same criteria discussed in the PER recommendation will be used to develop the hydraulic profile for the 0.25 mgd WWTP.

5.10 Solids Processing

PER Recommendation

As a part of the PER, various solids dewatering and solids handling options were evaluated based on capital costs, O&M costs and non-cost criteria. As a result of the evaluation, the recommended solids processing alternative included waste sludge holding tanks (WSHTs), dewatering, and composting followed by reuse/disposal of Class A biosolids which was determined to be most economical. A 1.0-meter belt filter press was recommended for the dewatering process. Tables 5.10-1 and 5.10-2 summarize the solids processing design criteria.

Table 5.10-1: PER Sludge Generation at Design Conditions

CONDITION	UNITS	PHASE I	PHASE II
Design Average (1) (2)	lbs/d	810	1620
Design Average	tons/million gallons	0.81	
Maximum Month (1) (2)	lbs/d	1,040	2,080
Maximum Month	tons/million gallons	1.04	
Volatile Content of Waste Sludge	%	80	80
Solids Concentration	%	0.5	0.5

Notes:
1. The sludge production estimated using BioWin® process modeling.
2. Assumes wasting from the biological process 8 hrs/day, 7 days/week.

Table 5.10-2: WSHTs Design Criteria

PARAMETER	UNITS	DESIGN CRITERIA	
		Phase I	Phase II
Minimum Storage			
At Design Average:	days	5	5
At Maximum Month:	days	3	3
Influent Solids Concentration	%	0.5	0.5
Effluent Solids Concentration(1)	%	0.5	0.5
Total Air Required(2)	scfm	400	800
No. of Blowers(5)	n/a	2(3)	3(4)
Total Volume Required	gallons	100,000	200,000
No. of Tanks	n/a	2	4
Volume/Tank	gallons	50,000	50,000

Notes:
1. No volatile solids destruction or decanting assumed in the WSHTs.
2. Based on air requirement of 30 scfm per 1,000 cf of WSHT volume.
3. 1-operational, 1-standby.
4. 2-operational, 1-standby.
5. Blowers are operated for 18hrs/day.

VE Modification

Composting was eliminated from the current construction project as a result of the VE session. The ability to truck dewatered sludge to a landfill was provided; however, the design will be able to accommodate composting in the future. In addition, the size and layout of the Solids Processing Building was optimized.

Current Direction

Sludge production has been reduced as a result of the reduced plant capacity. As a result, the volume of the WSHTs and capacity of the BFP have also been reduced. Tables 5.10-3 and 5.10-4 summarize the solids processing design criteria for the 0.25 mgd WWTP.

Table 5.10-3: 0.25 mgd Sludge Generation at Design Conditions

CONDITION	UNITS	PHASE I	PHASE II
Design Average (1) (2)	lbs/d	400	800
Design Average	tons/million gallons	0.81	
Maximum Month (1) (2)	lbs/d	500	1,000
Maximum Month	tons/million gallons	1.04	
Volatile Content of Waste Sludge	%	80	80
Solids Concentration	%	0.7	0.7

Notes:
1. The sludge production estimated using BioWin® process modeling.
2. Assumes wasting from the biological process 8 hrs/day, 7 days/week.

Table 5.10-4: 0.25 mgd WSHT Design Criteria

PARAMETER	UNITS	DESIGN CRITERIA	
		Phase I	Phase II
Minimum Storage			
At Design Average:	days	7	7
At Maximum Month:	days	5	5
Influent Solids Concentration	%	0.7	0.7
Effluent Solids Concentration(1)	%	1.0	1.0
Total Air Required(2)	scfm	240	480
No. of Blowers(5)	n/a	3(3)	5(4)
Total Volume Required	gallons	50,000	100,000
No. of Tanks	n/a	2	4
Volume/Tank	gallons	25,000	25,000

Notes:
1. Decanting assumed in the WSHTs.
2. Based on air requirement of 30 scfm per 1,000 cf of WSHT volume.
3. 2-operational, 1-standby.
4. 4-operational, 1-standby.
5. Blowers are operated for 18hrs/day.

6.0 Capital Cost Estimate

As noted in Chapter 9 of the PER, a total project cost of $30.8 million was estimated for the proposed 0.5 mgd WWTP including the MBR process and the composting facility. Through the final design process, the original project costs were refined, resulting in a total project cost estimate of $29.2 million. As a result of the VE session, the total project cost was reduced to $23.5 million based on the implementation of the VE recommendations.

As a result of the Town's decision to reduce the overall plant capacity to 0.25 mgd, a total project cost estimate was prepared for the reduced plant size. The anticipated cost for the current plant is summarized in Table 6-1.

Per Table 6-1, the new estimated project cost for the 0.25 mgd plant is $17.8 million.

Table 6-1: 0.25 mgd Facility Engineers Opinion of Probable Construction Costs

Description	Estimated Construction Cost	% Grant Eligible	$ Grant Eligible	$ Not Grant Eligible
Preliminary Treatment				
Headworks	$1,030,000	0%	$0	$1,030,000
Fine Screens	$559,000	40%	$223,600	$335,400
Emergency Overflow Tank	$385,000	0%	$0	$385,000
Biological Treatment				
Reactor Tanks and Equipment	$1,130,000	100.0%	$1,130,000	$0
Solids Separation Processes				
Membrane Process Tanks	$370,000	75%	$277,500	$92,500
Membrane Process Equipment	$1,860,000	75%	$1,395,000	$465,000
Process Building	$321,750	75%	$241,313	$80,438
Nitrate Recycle Pumping	$83,300	100%	$83,300	$0
Post Treatment				
UV Disinfection / Post Aeration / Effluent Flowmeter	$405,000	0%	$0	$405,000
Outfall Extension	$280,000	0%	$0	$280,000
Other Processes				
Methanol Feed System	$101,000	100%	$101,000	$0
Alum Feed System	$90,000	100%	$90,000	$0
Solids Processing				
Solids Processing Building	$770,000	48%	$369,600	$400,400
Waste Sludge Holding Tanks	$354,000	48%	$169,920	$184,080
Miscellaneous				
Demolish Existing Plant Structures		0%	$0	$0
Decommission and Demolish Existing Holding Pond		0%	$0	$0
Subtotal	**$7,739,050**	**53%**	**$4,081,000**	**$3,658,000**
Pro-Rated Items				
Operations Building	$393,300	15.29%	$60,145	$333,155
Plant Water System	$81,000	52.73%	$42,713	$38,287
Plant Recycle System	$76,000	52.73%	$40,077	$35,923
Yard Piping	$763,200	52.73%	$402,455	$360,745
General Site Work	$900,000	52.73%	$474,593	$425,407
Electrical Costs	$2,488,000	52.73%	$1,311,986	$1,176,014
Subtotal Construction Cost (Year 2009 Dollars)	**$12,440,600**	51.55%	$6,413,000	$6,028,000
Bonds, Insurance, Mobilization (7% Const. Total)	**$900,000**	51.55%	$463,941	$436,059

Contingency (10%)	**$1,100,000**	51.55%	$567,039	$532,961
Contingency (Membrane Equipment @ 10%)	**$190,000**	51.55%	$97,943	$92,057
Total Construction Cost (Year 2009 Dollars)	**$14,600,000**			
Preliminary Engineering	**$140,000**	51.55%	$72,169	$67,831
Design Engineering	**$1,540,000**	51.55%	$793,854	$746,146
Const. Admin, Insp., Town Admin, Prog (12%)	**$1,492,872**	51.55%	$769,560	$723,312
Total Project Cost (Year 2009 Dollars)	**$17,772,872**			
Net Construction Grant	**$6,410,000**			
Net Total Grant(1)	**$6,880,000**			
Town Contribution	**$10,890,000**			

Note:

1. **Assumes 75% grant funding.**

7.0 ATTACHMENTS

- Value Engineering Evaluation Submittal
- Value Engineering Submittal - including 30% design documents
- Preliminary Engineering Report